Appendix E
Tensile Strength of Roller Compacted Concrete[1]

The tensile strength of roller compacted concrete (RCC) differs from conventional concrete by the extent of material differences in the makeup of mixture proportions and the differences in the measures of control exercised in the production, methods of placement, and curing of the concrete. Any discussion of the tensile properties of RCC must also discuss the tensile properties of conventional concrete as well as other differences.

E-1. Tensile Properties of Conventional Concrete

a. Introduction. Raphael {1} discusses the tensile strength of concrete, the various test methods used for measurement, and the differences in test measurements. He also makes recommendations for relating tensile and compressive strengths in the design of concrete dams. While he discusses the importance of the tensile strength of concrete during earthquakes, he fails to consider any affects of the size of aggregate on tensile properties of massive dam concrete or to discuss the influence of factors other than surface drying on the tensile strength of cores. It is clear from the size of test specimens reported by Raphael that the vast majority of the 12,000 test specimens consisted of maximum aggregate sizes less than 2 in. The exception would be the 500 6-in. cores taken from the 14 concrete dams. No mention is made of size of aggregate or the possible influence of large aggregate, within the 6-in. core specimens, on test results.

b. Effects of mixture proportions and aggregate size on tensile strength. The tensile strength of concrete is dependent on the tensile properties of paste and aggregate, the bond of paste to aggregate, and the presence of any air voids and/or microcracking within the matrix. With normal weight aggregates, the bond of paste to aggregate generally controls the tensile strength of the concrete. Thomas and Slate {2} found the paste-aggregate tensile bond strength to vary from 41 to 91 percent of the tensile strength of paste depending on the rock type, the surface roughness of aggregate, and the w/c ratio. (Lowest values are sandstone aggregates and highest values are limestone aggregates.) They also found the mortar-aggregate tensile bond strength to vary from 33 to 67% of the tensile strength of mortar. Bond is enhanced by roughness of crushed aggregate surfaces and will improve with age of chemically reactive aggregates such as limestone. Bond may also be influenced by differences in thermal properties of aggregate and paste as a result of microcrack formation during cooling of the concrete from peak hydration temperatures. Such cracks, within the paste matrix, may be expected to heal with time; however, healing may not occur in bond to the aggregate.

(1) It is common practice in proportioning concrete mixtures for dams to utilize large aggregates in order to decrease cementitious material requirements and lower costs and heat generation. To effectively reduce the volume of paste required to coat all the aggregate particles and provide the workability required for placement, the total surface area of aggregate must be reduced by increasing the proportions of larger aggregate sizes. The effect of increased proportions of large aggregate on the tensile strength of concrete is to require a larger proportion of the tensile load to be transmitted through aggregate bond. The compressive strength of concrete is less dependent on aggregate bond than tensile strength. Thus, it is apparent that the relationship between the tensile strength of concrete and compressive strength of concrete not only varies with the method of test, as indicated by Raphael, but also varies with the type and maximum size of aggregate.

(2) Test data on the tensile properties of conventional mixtures containing aggregate larger than 1-1/2 in. is limited. Walker and Bloem {3} tested aggregates from 3/8 to 2-1/2 in. in 6- by 12-in. cylinders but found little effect of aggregate size on splitting tensile strength with the natural gravel tested. Tynes {4} compared the splitting tensile strength of 6-in. limestone aggregate mixtures in 20- by 40-in. cylinders with wet-screened 6- by 12-in. cylinders. The effect of excluding aggregates larger than 1-1/2 in. from the test specimens is indicated by the variation in tensile strength ratio of small to large specimens from 1.25 to 1.36. By comparison, the variation in compressive strength ratio of small to large specimens was from 0.99 to 1.15.

[1] Robert W. Cannon, Consulting Engineer.

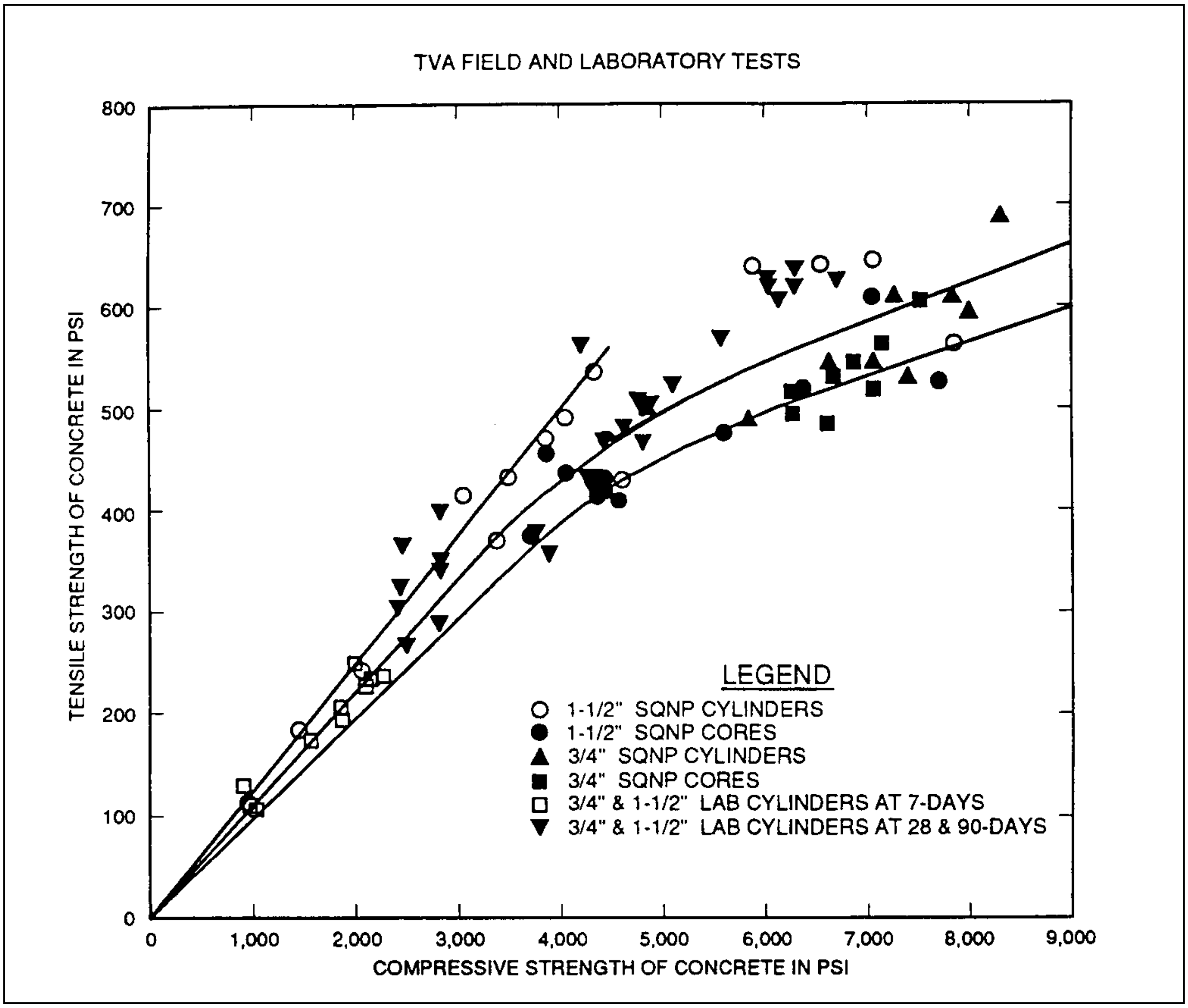


Figure E-1. Tensile strength versus compressive strength for conventional concrete

(3) Figure E-1, is a plot of data taken from studies performed at TVA's Singleton Laboratory and TVA's Sequoyah Nuclear Plant on comparison of splitting tensile and compressive strengths with limestone aggregates having a maximum size of 3/4 and 1-1/2 in. in mixtures containing fly ash and for strengths at ages varying from 7 days to 2 years. Compressive strengths varied from 900 psi to 9,000 psi. Laboratory specimens were standard-cured to the time of test. Field cylinders were standard-cured for 28 days, then sealed in plastic bags and stored at standard temperatures until tested. The cores were drilled from five 6-ft-high by 6-ft-wide by 2-ft-thick blocks at 90 days age, then sealed in plastic bags and stored at laboratory temperatures until tested.

(4) In comparing the ratios of individual tensile strength tests with compressive strength, or with compressive strength raised to the powers of 1/2 or 2/3, it is obvious that the data do not fit any single equation. The only obvious trend is the changing of ratios with strength. Figure E-2 is a plot of the data from reference {3} which shows the same basic trends.

(5) Prediction equations, based on variation with compressive strength raised to the powers of 1/2 or 2/3, are reasonably accurate in predicting tensile strengths of structural concrete in excess of 3,500 psi but overpredict tensile strengths with lower strength concretes. For compressive strengths less than

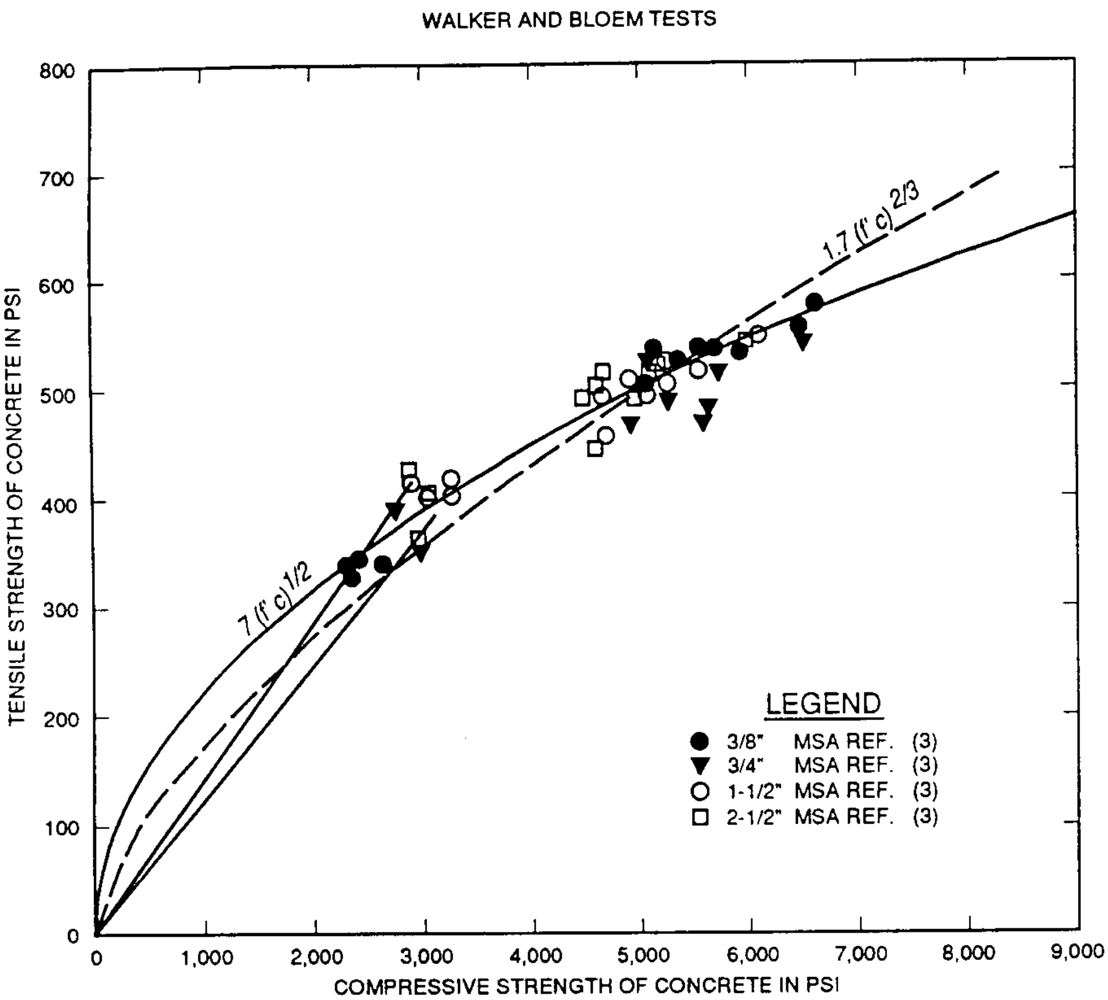


Figure E-2. Tensile strength versus compressive strength for conventional concrete

3,500 psi, tensile strengths vary in direct proportion to compressive strengths.

(6) Figure E-3 from reference {2} shows that the tensile bond strength of mortar to aggregate is relatively unaffected by changes in w/c ratio from 0.36 to 0.75 whereas the tensile strength of the mortar is significantly affected. It also shows that increasing the sand fraction by increasing the cement/sand ratios of the mortar from 1:2 to 1:3 lowers bond strength more than increasing the w/c ratios. From this it would appear that the proportion of tensile strength transmitted through bond is higher for lower strength concretes and lower for higher strength concretes. This would explain the changing relationship of tensile properties with strength.

c. Splitting tensile versus direct tension tests.

(1) Splitting tensile tests are simple to perform and generally have reasonably low within-test-variations similar to that of compression tests. On the other hand, load transfer has always been a problem in direct tension tests. Epoxied end plates have been used to transfer direct tensile loads but clean-up of plates after testing is a major problem. In some cases strengths are also limited by bond of the epoxy to the specimen. Epoxy bond failure was experienced

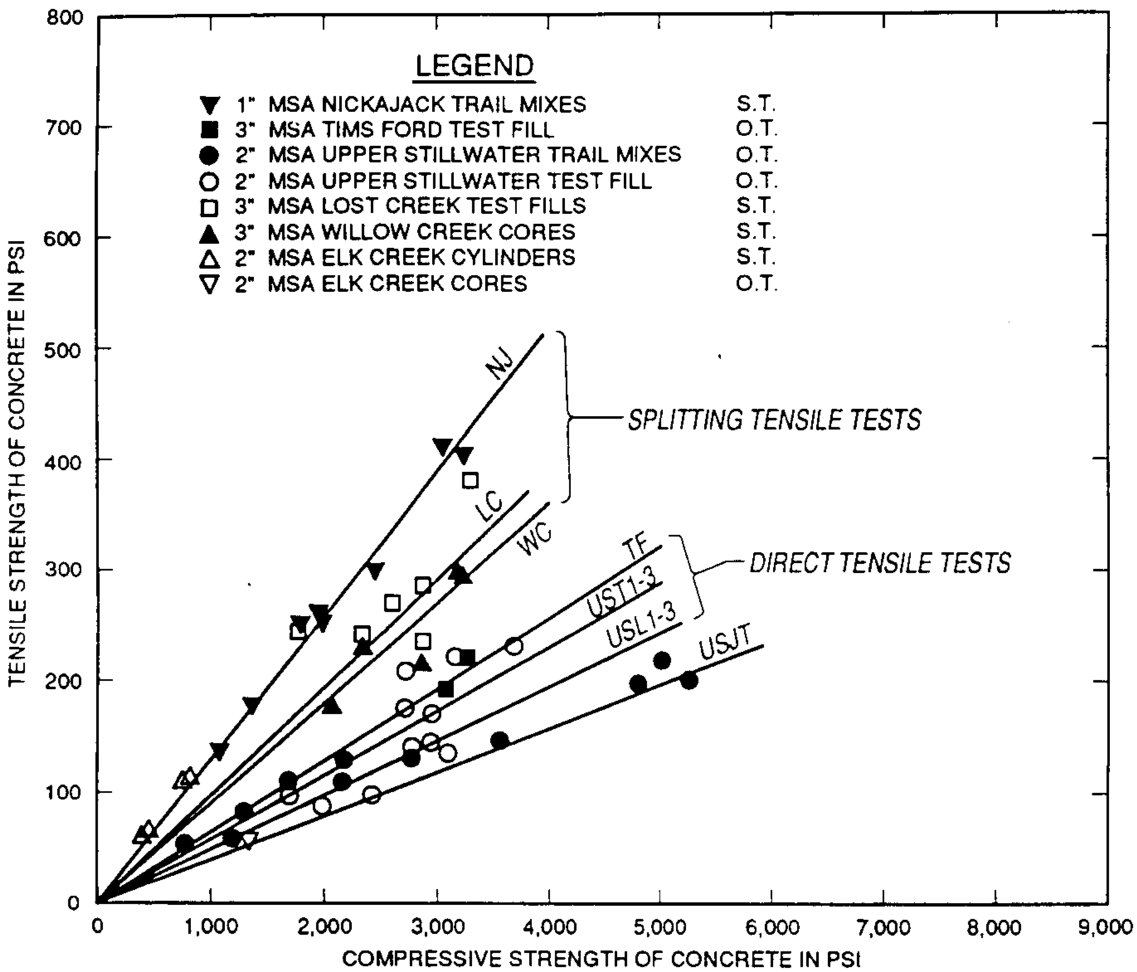
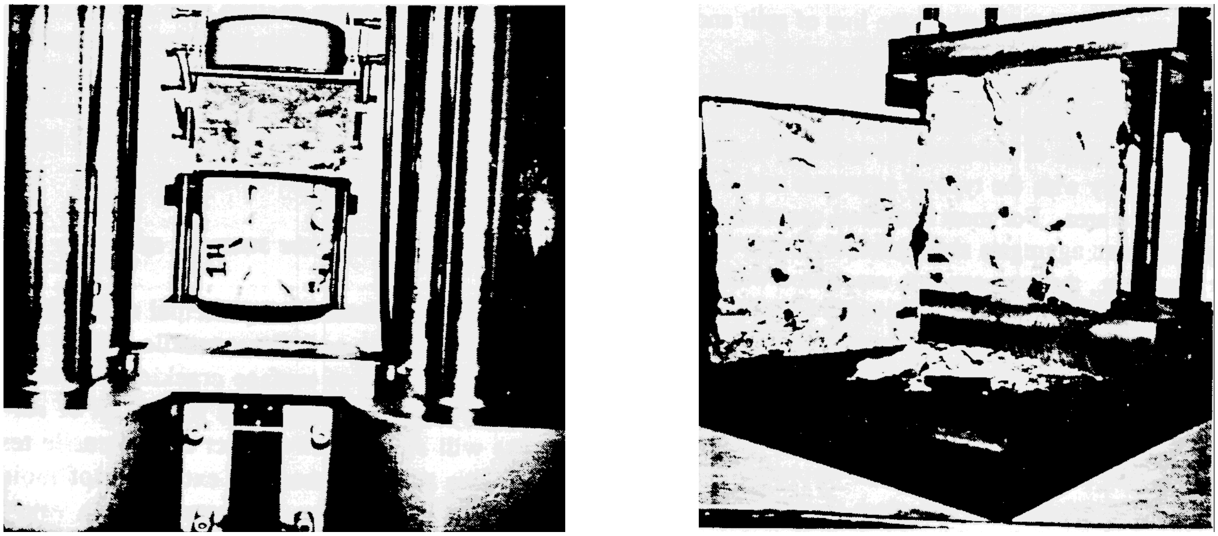


Figure E-3. Tensile strength versus compressive strength for roller compacted concrete

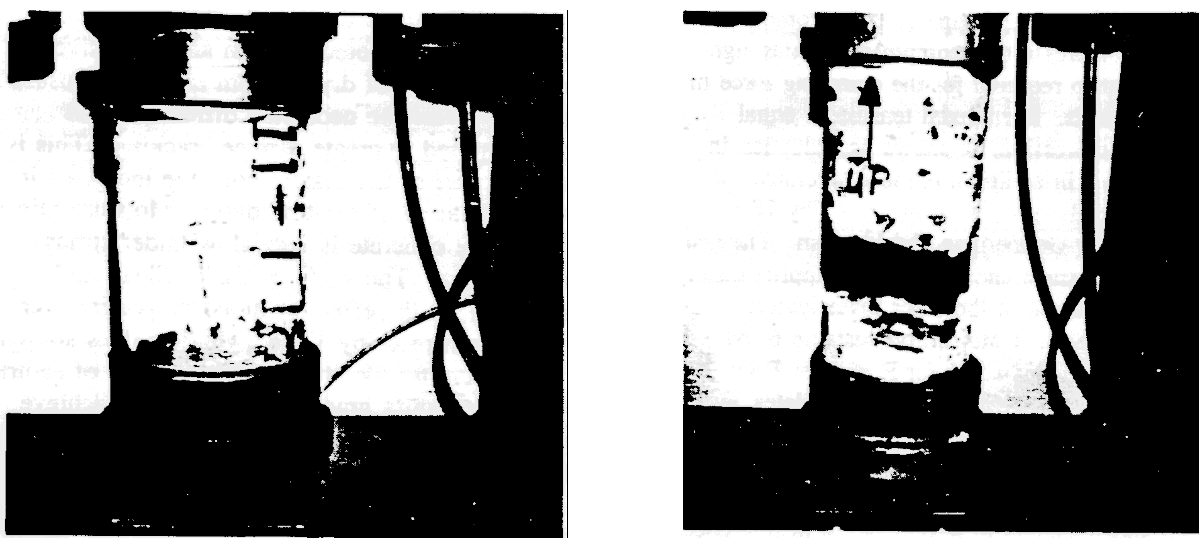
at approximately 280 psi tension in reference {7} tests. A simple means of performing the direct tensile tests is shown in Figure E-4 using standard capping compound and 2-in. deep socket end plates for applying load. These were developed for use at Tims Ford {16}. They have been used for maximum tensile strengths as high as 345 psi without failure of the connection.

(2) Direct tension test results are lower and somewhat more variable than splitting tensile tests. Because of the problems involved with the direct tension test, most investigators accept the splitting tensile test as being representative of concrete tensile strengths. However, there are some distinct differences in the two tests which account for the differences in results and should be considered in evaluating tensile strengths.

(3) Dunstan {9} discussed the anisotropic nature of concrete and the effects on strength of the orientation of testing with respect to the orientation of casting. Most of his discussed research involved casting and testing the compressive strength of cubes and prisms in the horizontal and vertical positions. Cubes tested with the axis of casting vertical are reported {10, 11} to be stronger in compression by about 12% to 15% than those tested with the axis of casting



a. Push-off test for shear strength



b. Tensile strength test

Figure E-4. Strength tests

horizontal. Similarly, prisms were reported {11, 12} to be 8% and 18% stronger cast and tested vertically while a value of 12% was reported {13} for cores. The behavior of concrete in tension is less documented; however, one report {12} found the strength of specimens tested with the axis of casting vertical to be 8% weaker than that of samples tested with the axis of casting horizontal. Bleed water accumulation at the underside of aggregate particles was the reason given {14, 15} for this anisotropic behavior in conventional concrete. In the case of cubes and prisms there is also a difference in efficiency of tamping specimens in the vertical and horizontal directions which contributes to the differences in test results since all testing was done in the vertical plane. In the Tims Ford test fill {16}, the compressive strength of horizontal cores was 27% higher than that of vertical cores which is exactly opposite to the findings in reference {13}.

(4) In the splitting tensile test the plane of failure is normally in line with the direction of casting. Failure is restricted principally to the line of split and goes through aggregate as well as mortar with the amount of failed aggregate increasing with compressive strength. If aggregate bond is the weak link in the tensile properties of concrete, the splitting tensile strength will vary with the type of aggregate and bonded surface area. For most structural concrete having maximum aggregate sizes generally equal to or less than 1 in., the over-strength associated with the controlled plane of failure is probably in the order of 10% to 15% depending on aggregate shape and w/c ratio. With larger size aggregate, the difference may be substantially more.

(5) Raphael plots the direct tensile results of Gonnerman and Shuman {5} but gives them no further consideration because of the friction grips at the ends of the 6- by 18-in. test specimens. He quotes Rusch {6} as the basis for lower tensile strengths. Rusch does show reduced capacities for combined tension and compression at compression loads significantly higher than required for the clamping force in direct tension tests. For biaxial tension or equal tension and compression, he shows no reduction in tensile capacity. In addition, the test specimens of Rusch are radically different from the 6- by 18-in. cylinders used by Gonnerman and Shuman. The test results for Gonnerman and Shuman are approximately 20% lower than those of the other investigators reported by Raphael. Comparative tests on 6- by 12-in. wet screened cylinders for Portugues Dam {7}, for direct tension tests with epoxied end plates, averaged 0.8 of the corresponding splitting tensile tests with comparisons ranging from 0.7 to 0.92. The average ratio of direct to split tensile strength for 6-in. aggregate concrete in Report No. 4 of reference {24} was 0.77. It thus appears that a 20% reduction may well represent the difference between splitting tensile and direct tensile tests for the aggregate sizes investigated by Raphael.

(6) If the concrete within a direct tension test specimen is uniform throughout, failure will occur in the central portions of test specimens having length-to-diameter ratios equal to or greater than two. If the compression of friction clamps at the specimen ends affects a reduced tensile capacity as Raphael indicates, failure would occur at the ends and not in the interior of the test specimen. Friction grips are not as previously discussed. Direct tension test failures typically fail bond around the aggregate and always occur at the weakest cross section of the specimen. Thus failure is associated with both the weakest axis and the weakest plane in the axis. Therefore the average of direct tensile test results may be assumed to represent the minimum tensile properties of the concrete.

d. Factors affecting the strength of cores.

(1) Raphael attributed the principal differences in splitting tensile testing and direct tensile testing of cores to the formation of surface cracking due to differential drying shrinking. The formation of surface cracks will significantly affect direct tensile test strengths. It is apparent that any extraction of moisture from the surface of the concrete leaves a void within the matrix which will affect strengths to some degree. Some investigations disagree with his assumptions concerning the magnitude of restraint created by differential drying. In the first place, cores are normally protected from any extensive drying and the rate of drying from normal exposure is too slow to create the necessary differentials in restraint required to create surface cracking. This is particularly true of the mature concrete indicated in the re-examination of existing dams. Moisture migration in mature concrete is very slow under atmospheric exposure. The study of Cady, Clear, and Marshall {8} on the effect of moisture gradients on tensile strength of 6- by 12-in. cylinders does show a loss of splitting tensile strength with degree of saturation due to moisture gradients. In order to achieve those gradients their tests were rapidly performed “in a vented, air-circulating oven at either 187 C or 110 C for sufficient lengths of time to produce different degrees of saturation.”

(2) In the TVA Sequoyah N. P. study, the curing for both cores and cylinders from 90 days to 2 years age was identical. In comparing the relationship of split tensile to compressive strengths at 90, 180, 365, and 730 days for the five different strength concretes, the variation of tensile strength to the square root of compressive strength was more uniform than either a direct comparison or comparison with compressive strength to the 2/3 power. The following tabulation summarizes the average of the tests at the indicated ages:

1-1/2-in. MSA				**3/4-in. MSA**			
Compressive Strength, psi		Tensile Strength, psi		Compressive Strength, psi		Tensile Strength, psi	
Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores
4,900	5,160	521	471	7,190	6,840	577	526

The data indicate 9% lower core strengths for the 3/4-in. and 10% lower core strengths for the 1-1/2-in. aggregate indicating a probable minimum effect of coring on split cylinder strengths.

(3) In the extraction of cores, the surface of the core is subjected to a variety of strains due to the effects of the differential hardness of paste and aggregates on the cutting action of the core bit, the expansion of the surface relative to the interior of the core due the sudden stress relief by the cutting action, and the torque imposed on the core by the rotating bit. The successful drilling of cores requires an experienced operator. It also requires a double-barrel core bit which has an inner barrel to clamp and hold the core to reduce the torque imposed on the core and to reduce breakage and loss of core at weak sections. Successful extraction also depends on the strength of concrete and is probably more dependent on tensile than compressive strengths. All of these factors contribute to surface defects which act as stress raiser or crack initiators from which a crack can propagate at a stress lower than the tensile strength of the material. Once a crack is initiated it will propagate under a lesser load than required to initiate the crack. The relative magnitude of propagating crack load has not been quantified; however, Raphael's 50% reduction for direct tensile strength of cores extracted from dams may be indicative of this effect with large size aggregate.

(4) In the splitting tensile test the orientation of surface cracks due to the coring operation is at right angles to the splitting failure plane and therefore relatively unaffected by the orientation of cracks compared with the in-line orientation in the direct tensile test. Past experience has shown that core extraction is sensitive to core diameter. It is not unusual to obtain complete core recovery with a larger core where problems of core recovery are encountered with a smaller one. It also appears reasonable to expect the ratio of aggregate size to core size to affect test results.

(5) Direct tensile testing of vertical cores should be used in determining the tensile properties of horizontal construction joints or of concrete in the vertical direction. Point load testing is a splitting test performed on the cross section of cores or cylinders and may also be used to determine the strength of horizontal construction joints from vertical cores. Split tensile testing of horizontal cores has been used to establish joint strength; however, identification and location of the joint in the central portion of the core, for correct performance of the test, is very difficult. Core recovery of weak joints is generally more successful if extracted at an angle rather than vertical; however, tests performed on cores extracted in any other plane may overestimate tensile strengths on the horizontal plane considering the anisotropic nature of concrete.

(6) In the above-mentioned TVA field tests with the 3/4- and 1-1/2-in. aggregates, curing was identical following the extraction of cores; therefore, the apparent 10% reduction in splitting tensile strength of cores over cylinders was not affected by differential drying. For direct tensile testing of cores, the minimum effect of vertical core extraction would be more than 10% and probably less than Raphael's 50%.

e. Tensile strength of conventional mass concrete.

(1) The minimum design tensile strength for static analysis should be based on the direct tensile strength of the concrete. The relationship between direct tensile strength and compressive strength may be determined from known splitting tensile strengths by the following:

(2) For compressive strengths less than 3,000 psi:

The tensile splitting strength of 6- by 12-in. wet-screened cylinders containing 1-1/2-in. and smaller size aggregates may be expected to vary from 0.10 f'_c to 0.15 f'_c depending on type of aggregate.

(3) For compressive strengths equal to or greater than 3,000 psi:

For tensile splitting strength of 1-1/2-in. and smaller size aggregate use one of the following formulas with an expected range of plus or minus 15% depending on aggregate type:

$f'_{st} = 1.7\ (f'_c)^{2/3}$ Raphael's formula

$f'_{st} = 7\ (f'_c)^{1/2}$

(4) For aggregates larger than 1-1/2 in., reduce strengths by 10%.

For direct tensile strengths, reduce strengths by an additional 20%.

E-2. Tensile Properties of Roller Compacted Concrete

a. Introduction. The definitions of terms "roller compaction" and "roller compacted concrete" in ACI 207.5R {17} are broad definitions which can be applied to almost any mixture of materials containing cement and having sufficient stiffness to support any type of roller during compaction. Thus RCC mixtures can vary anywhere between that of a dense, high quality concrete to that of a porous, low quality cemented conglomeration of aggregate particles. The tensile properties of RCC may therefore be expected to vary widely.

b. Establish basic mix. In establishing the basic mix for RCC, the needed stiffness for support of rollers during compaction is best accomplished by establishing the coarse aggregate fraction slightly higher than that of conventional concrete having the same maximum size aggregate. (See Table 2.2 of reference {17}.) The mortar fraction of the mixture should be proportioned to provide the strength requirements of the mixture and the workability needed for uniform compaction and consolidation during placement. Workability, as a measure of vibration time, is affected by the maximum size, quantity, and grading of coarse aggregate and the makeup of the mortar fraction. The utilization of any specification requirement which increases the water requirements, of the mortar fraction, is detrimental to proportioning efficiency. The practice of increasing the fines content of the fine aggregate may be beneficial in the compaction of unworkable mixtures, but simply requires an increase in paste to maintain workability at a given level for workable mixtures. For a given makeup of mortar, the optimum coarse aggregate fraction is the maximum providing the desired level of workability.

c. Consistency and workability.

(1) The Elk Creek test fills {18} clearly demonstrated that lean 3-in. MSA mass RCC concrete should have a minimum workability in the range of 10 to 20 seconds vibration time when measured by the modified vebe test method using a 27.5-pound surcharge. As a result of that experience, mixtures with vibration times in excess of 30 seconds are not recommended. It has been this author's experience that a 20- to 30-second time frame is optimum for 1-1/2-in. and smaller MSA mixtures without a surcharge using the TVA test procedure {19} of a loosely filled unit weight container filled to the top.

(2) Dunstan {20} found that the TVA procedure and the standard vebe had essentially the same vibration times for 1-1/2-in. MSA mixes with vibration times equal to or less than 30 seconds. In the Lost Creek RCC investigations {21}, laboratory studies indicated that the standard vebe required approximately twice the vibration time (35 sec) compared with that of using a 27.5-pound surcharge (17 sec) for a 3-in. MSA mix with 160 pounds of water. In the Upper Stillwater laboratory investigations {21}, a 50-pound surcharge was used to modify the vebe. For the wetter mixes the modified vebe required vibration times of 25 to 35 seconds compared to vibration times of 35 to 45 seconds without the surcharge. By either of the above criteria, the Upper Stillwater laboratory mixes appear to be in a questionable range of workability. This is verified by the adjustments to the RCC mix at Upper Stillwater from an average vebe time of 30 seconds to about 17 seconds following the 1986 coring program {27}. USBR personnel indicate that future RCC designs will have a still lower target vibration time.

(3) It should be apparent from the above discussion that the trends of all agencies concerned with the quality of concrete placement are toward increased workability and shorter vibration times. It is also apparent that a range of plus or minus 5 seconds is needed for reasonable control of consistency. The

surcharge weights of 27.5 and 50 pounds by the Corps and Bureau were established on the basis of substantially less workable mixes. The use of the surcharge may be an unnecessary hindrance to the performance of the test if the workability of the concrete can be controlled within a 10-second time frame without the surcharge.

d. Effect of consolidation and compaction on tensile properties.

(1) Conventional concrete is deposited in piles or within forms in a loose configuration which is then broken down and consolidated by internal vibration. If the mixture is properly proportioned there will be little if any separation or segregation of the coarse aggregate particles from the matrix of mortar during the consolidation process with the larger aggregate particles remaining suspended within the matrix of the mortar. Thus the particles of coarse aggregate remain in a generally random orientation irrespective of their particle shape.

(2) RCC is deposited in piles and spread in a loose configuration with a dozer or similar piece of equipment. During the spreading operation, the coarse aggregate particles are not suspended within the matrix of the mortar and temporary separation of aggregate and mortar occurs. During this temporary separation, flatter particles tend to align their flatter sides with the horizontal {9}. If the mortar matrix has poor workability or is insensitive to vibration no lateral movement of coarse aggregate particles will occur during compaction. If the mortar matrix is workable and sensitive to vibration, lateral displacement and consolidation of the coarse aggregate particles will occur resulting in a more random orientation of the coarse aggregate particles. The extent of this occurrence and its effect on tensile properties of the concrete is dependent on the particle shape of the aggregate.

e. Anisotropic nature of RCC.

(1) The water content of RCC is generally assumed to be less than that of conventional concrete because of its no-slump consistency. This may, or may not, be true dependent on the aggregates and the proportioning of the mortar matrix for which the comparison is made. Thus RCC may be as susceptible to the accumulation of water under the aggregate particles as conventional concrete. The anisotropic nature of the tensile strength of RCC may equal or exceed that of conventional concrete dependent on the shape and gradation of the aggregates, their affect on water contents, and the extent to which the flatter particles increase the surface area of the aggregates on the horizontal plane. Thus the magnitude of differences in the tensile strength of RCC, on horizontal planes compared with vertical or other planes within the concrete, will vary with mixture proportions, type, size, and shape of the coarse aggregate.

f. Data on RCC tensile strengths.

(1) Figure E-3 is a plot of split cylinder and direct tensile tests for RCC. Only a limited amount of both split cylinder and direct tensile tests were performed on the same concrete in these plots; however, the difference in test methods is apparent. Additional data were obtained from ASCE special publication “Roller Compacted Concrete II” {27, 28, 29, 30}.

(2) At Galesville {27}, the ratio (ST/C) of split tensile strength to compressive strength was 0.134 and the ratio of direct tensile to split tensile (DT/ST) of untreated construction joints was 0.24. Where bedding concrete was used the ratio was 0.5.

(3) At Upper Stillwater {21}, the average (DT/C) ratio of cylinders for Mixes L-1 through L-3 was 0.050 whereas the average (DT/C) ratio of parent core material from test fill mixes T-1 through T-3 was 0.058. Apparently the coring of the relatively soft sandstone aggregate concrete had little, if any, affect on strengths. In the 1986-87 coring program for the dam {27}, the (ST/C) ratio of cores from the dam was 0.076 and the (DT/ST) ratio of intact construction joints was 0.5. While there was substantial increase in tensile strengths from one to two years age, there was very little, if any, change in the (DT/C) ratios.

(4) The average (ST/C) ratio for concrete from reference 30 for six different mixes having relatively high water contents (220 lb to 260 lb) was 0.167.

(5) For the same size and type of aggregate, the direct tensile strengths of RCC appear to be 25 to 30 percent lower than splitting tensile strengths. This compares with a 20% reduction for conventional concrete. While the data are limited, the range of splitting tensile strengths of RCC appears to correspond to that of conventional concrete.

(6) The data on tensile strength from Elk Creek Dam are limited and difficult to interpret. In the test fills of 1982 and 1985, the principal interest was the shear strength of joints from sawed blocks out of the test fills and cores were not extracted. The results of split cylinder tests at 28 and 90 days age are shown in Figure E-3 and indicate a (ST/C) strength ratio of approximately 0.15. The average (ST/C) ratio of cores extracted from the dam was 0.17. In contrast to this, the results of direct tensile tests on cores, extracted from the dam concrete, indicate a (DT/C) ratio of approximately 0.04. This difference is unrealistic in comparison with other test data and must be attributed to the effects of coring the relative low (1,300 psi) strength concrete. (Observation of the cores indicates substantially higher breakage than should have occurred in a quality coring operation. Apparently a double barrel core bit was not used and the contractor was more interested in production than quality.) As a minimum, the direct tensile strength of the parent concrete should not have been less than 1/2 of the splitting tensile strength.

g. Lift joints.

(1) The critical tensile properties of concrete for seismic resistance to earthquakes are the lift joints. It is not reasonable to expect the bonding of cold joints, under varying conditions of exposure, to be equivalent to that of the parent material. From 1959 through 1973 the U.S. Army Engineer Waterways Experiment Station (WES) {24} investigated methods of treating horizontal construction joints for conventional mass concrete. All joints were standard cured. From Report No. 1, uncleaned joints averaged approximately 2/3 of cleaned joint strengths. After completion of all tests, Report No. 4 concluded the method of treatment (wet, dry, mortar, no mortar) was judged to have no significant bearing on strength.

(2) WES {25} also performed laboratory tests on the bonding of workable 1-1/2-in. MSA RCC using a relatively small roller and 6-in. lifts. In one investigation the untreated, moist-cured joints exposed for 1 hour and 24 hours had joint tensile strengths of 87 and 53% respectively, of parent concrete. (Note: This comparison is intended to show the relative effects of joint age on strengths. It does not suggest equivalent compaction of the small roller, nor equivalent strengths, to normal RCC placement.) In another investigation the relative joint strengths of lean and rich RCC mixtures along with conventional concrete indicated joint strengths of 27, 44, and 74%, respectively, of parent concrete for 1-day-old untreated joints. (Please note the similar results of conventional concrete with the earlier investigations.) In these tests it is important to note that only 7 of 12 joints of the lean mixture bonded compared with 11 of 12 for the richer mix and 100% for the conventional mix.

(3) The condition of the joint surface is critical to the development of bond regardless of the covering mixture. Plastic concrete is particularly vulnerable to drying during the hardening process when subjected to low humidity, warm weather, and/or windy conditions. There is sufficient moisture within the concrete itself for continued hydration if the moisture is not removed by drying. If the rate of drying is slow, the migration of moisture to the surface from below is sufficient for continued hydration and strength development of the concrete at the surface. If rapid drying occurs, the hydration process at the surface will stop along with any development of strength. It is thus critical, for bond development, to protect the plastic concrete from rapid drying during setting of the concrete. Under conditions of rapid surface drying, it may be necessary to cover the RCC for the first two to three hours after placement until the concrete reaches initial set. After initial set, water from a mist spray should not be expected to affect the w/c ratio of the surface concrete {26}.

(4) Surface conditions are also affected by the size of aggregate and the workability of the concrete. If the concrete is workable, paste and mortar will rise to the surface under the vibrating action of the roller and the finished surface of the concrete will be relatively smooth if not over-rolled. (Over-rolling is evident when paste accumulates on the roller and mortar is picked up from the surface.) If the concrete is unworkable, there will be little if any lateral displacement of the concrete during compaction and paste and mortar will not rise to the surface. The surface will generally be rough and granular in texture as mortar and aggregate are simply forced into a closer relationship of their spread condition by the compactive effort.

(5) A principal difference between workable and unworkable concrete, with respect to bond, concerns the compactive density of the concrete with respect to the hardened concrete surface. With workable concrete, the density of the covering mixture increases with depth becoming a maximum at the

hardened surface {16}. With unworkable concrete the reverse occurs.

(6) Bond is dependent upon the intimate relationship of the covering mixture to the hardened surface. Maximum bond has always been achieved with sand blasting which removes a minimum of material and leaves the surface in a relatively smooth condition. Thus bonding improves with relative smooth hardened surfaces allowing lateral displacement of aggregates at the surface and densification of the workable concrete. It should therefore be apparent that the opposite effect can be expected with rough surfaces and unworkable concrete.

(7) Large aggregates decrease workability, increase surface roughness, and create voids at the surface due to bridging of large particles. All testing to date indicates that bond will decrease with increased size of aggregate. For bonding of maximum size aggregates larger than 1-1/2 in., it is essential that a bedding mix of mortar or slumpable concrete of 1 in. and smaller size aggregate be used {22}.

(8) Some idea of the relative bonding qualities of workable and unworkable concrete can be seen from examination of cores and data from Willow Creek and Elk Creek Dams {23}. At Willow Creek, consistency measurements of vibration time were considered unsuitable for controlling water contents. In the direct tensile testing of 9-in. cores at Willow Creek, the average strength of bonded joints was 46% of the parent concrete. (Limited data on 6-in. cores indicate joint strengths in excess of parent concrete which is unreasonable.) The percentage of bonded area, based on examination of cores, would indicate from 30 to 50% bonded. At Elk Creek, consistency measurements were used to control the workability of the concrete. The combined effectiveness of workable concrete and the use of mortar on cold joints is indicated by an apparent joint efficiency of 82%. While the data for both joint and parent concrete are below probable strengths, as previously discussed, their relative values should be indicative of actual conditions. A 70% intact joint recovery and only 15% smooth joint separations indicating a possible 85% bonded joints was reported at Elk Creek.

(9) In comparing the effects of cementitious quantities of mixtures using natural river gravel (supplemented by crushing) and requiring the same vebe time for compaction on bond strength, the bond strength of mixtures containing 150 pounds of cement is improved from 6.3 to 9.2% of their compressive strengths with joint treatment, whereas the bond strength of mixtures containing an additional 150 pounds of fly ash is improved from 6.8 to 7.4% with joint treatment {28}. From the same report, similar cementitious mixtures using limestone aggregates had direct tensile core strengths for the lean and richer mixes of 9.5 and 7.8% of their compressive strengths which were 840 and 1,920 psi, respectively.

h. Joint treatment and clean-up.

(1) One of the expressed advantages of RCC construction has been the elimination of the need for joint clean-up (if uncontaminated by foreign substances) because of the lack of laitance. The 82% bonded joint efficiency at Elk Creek compared with the 74% joint efficiency of conventional concrete on one-day-old joints in the WES {25} experiments may be considered an indication of the laitance effect. This advantage for RCC is dependent on strict adherence to specification requirements for curing and joint treatment, otherwise there is an increased probability of joint contamination and improper curing because of the increased number of cold joints in RCC construction.

(2) The relative strength of clean and treated joints for conventional mass concrete, in the WES {24} experiments, was approximately 70% of the parent concrete. At Elk Creek, joints were not cleaned unless contaminated or more than 72 hours old. The possible 85% bonded joints for Elk Creek represents a combined efficiency of bonded area and joint efficiency which is comparable to that of cleaned and treated joints for conventional construction.

(3) While the results of the lean and richer mixtures of workable RCC in the WES investigations may be lower than expectations due to differences in laboratory and field compaction, their differences are a clear indication of the effect of cementitious material contents on the bonding of cold joints. Thus where bedding mixes of either concrete or mortar are used to achieve or improve bond, they should be proportioned for higher strengths than the strength of the cold joint concrete.

(4) Extensive clean-up of cold joints is not required for development of bond with RCC as long as the joints are not contaminated by foreign

substance and are maintained in a relative moist condition throughout exposure. All loose particles of aggregate or mortar should be removed by washing with low pressure just prior to placement of covering concrete.

i. Tensile strength of RCC lift joints.

(1) For workable mixtures, the tensile bond strength of properly clean and cured joints covered with a suitable mortar or bedding mix may be assumed as 70% of the tensile strength of parent material which is equivalent to the joint strength of properly prepared conventional concrete.

(2) The tensile strength of parent material should be based on direct tensile test strengths or a maximum of 75% of splitting tensile strengths. If test strengths are based on wet-screening of aggregates larger than 1-1/2 in., reduce test values by 10%.

(3) When test data are not available, the following represents a range of acceptable design values for workable mixtures based on type of aggregate. Low values are based on natural aggregates and sandstone and the high values are based on all crushed aggregates such as limestone.

(a) For design compressive strengths less than 3,500 psi: The splitting tensile strength for RCC mixtures containing aggregates smaller than 2 in. may be expected to vary from 0.08 f'_c to 0.17 f'_c. For 1-1/2-in. and smaller MSA, the direct tensile strength of RCC lift joints may be assumed to range from 0.04 f'_c to 0.09 f'_c.

(b) For design compressive strengths equal to or greater than 3,500 psi: The splitting tensile strength for RCC mixtures containing aggregates smaller than 2 in. may be expected to vary from 5.5 to 8.5 times the square root of f'_c. For 1-1/2-in. and smaller MSA, the direct tensile strength of RCC lift joints may be expected to range from 3.0 to 4.5 times the square root of f'_c.

(c) For aggregates larger than 1-1/2 in., reduce these values by 10%.

(4) For unworkable mixtures which will not consolidate within 30 seconds vibration time:

(a) If mortar is used on all lift joints, use 2/3 of the design tensile strengths for workable concrete.

(b) If mortar is not used, use 1/3 of the design tensile strengths for workable concrete.

j. Design tensile strengths for dynamic and finite element analysis.

(1) Raphael {1} discusses the effects of dynamic loading on the tensile strength of concrete and the effects of nonlinear strain at failure on finite element analysis.

(2) For static finite element analysis, increase the above design tensile strengths of lift joints by a factor of 1.35.

(3) For seismic finite element analysis, increase the above design tensile strengths of lift joints by a factor of 2.

E-3. Tensile Strengths of Conventional and Roller Compacted Concrete.

Split and design tensile strengths of conventional mass concrete and roller compacted concrete are shown in Tables E-1 through E-3.

E-4. Conclusions and Recommendations

a. The design tensile strength of concrete should be based on the direct tensile strength of concrete in the direction of principal stress and should account for the relative strengths of construction joints and the effects of construction methods on the probability of attaining anticipated joint strengths.

b. The relationship between the direct tensile strength of RCC and conventional mass concrete varies with the workability of the RCC mixture and the quality of the concrete placement. The trends toward more workable RCC mixtures and improvements in production control are indicative of tensile strength attainment equivalent or superior to the conventional concrete.

c. The bond strength of horizontal construction joints depends on: (1) the soundness of the cold joint, (2) the paste content and strength of the covering mixture, and (3) the amount of reflective vibration and compaction at the joint.

Table E1
Conventional Mass Concrete

MSA in.	Max/Min	Split Tensile Strength[a] ≤ 3.0 ksi	Split Tensile Strength[a] > 3.0 ksi	Conv. Fctr.[b]	Design Tensile Strength[c] ≤ 3.0 ksi	Design Tensile Strength[c] > 3.0 ksi
≤ 1.5	Max	$0.15\ f'_c$	$8\ (f'_c)^{1/2}$	0.56	$0.085\ f'_c$	$4.5\ (f'_c)^{1/2}$
	Min	$0.10\ f'_c$	$6\ (f'_c)^{1/2}$	0.56	$0.055\ f'_c$	$3.4\ (f'_c)^{1/2}$
> 1.5	Max	$0.15\ f'_c$	$8\ (f'_c)^{1/2}$	0.50	$0.075\ f'_c$	$4.0\ (f'_c)^{1/2}$
	Min	$0.10\ f'_c$	$6\ (f'_c)^{1/2}$	0.50	$0.050\ f'_c$	$3.0\ (f'_c)^{1/2}$

[a] Splitting tensile strength of parent material.
[b] Includes conversion for direct tensile, joint strength, and probable percent of bonded joint.
[c] Direct tensile strength of construction joints.

Table E2
Roller Compacted Concrete, Consistency ≤ 30 Seconds Vibration

MSA in.	Mortar Yes	Mortar No	Max/Min	Split Tensile Strength[a] ≤ 3.5 ksi	Split Tensile Strength[a] > 3.5 ksi	Conv. Fctr.[b]	Design Tensile Strength[c] ≤ 3.5 ksi	Design Tensile Strength[c] > 3.5 ksi
≤ 1.5	-	-	Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.53	$0.090\ f'_c$	$4.5\ (f'_c)^{1/2}$
			Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.53	$0.040\ f'_c$	$2.9\ (f'_c)^{1/2}$
> 1.5	Y		Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.47	$0.080\ f'_c$	$4.0\ (f'_c)^{1/2}$
	Y		Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.47	$0.040\ f'_c$	$2.6\ (f'_c)^{1/2}$

[a] Splitting tensile strength of parent material.
[b] Includes conversion for direct tensile, joint strength, and probable percent of bonded joint.
[c] Direct tensile strength of construction joints.

Table E3
Roller Compacted Concrete, Consistency > 30 Seconds Vibration

MSA in.	Mortar Yes	Mortar No	Max/Min	Split Tensile Strength[a] ≤ 3.5 ksi	Split Tensile Strength[a] > 3.5 ksi	Conv. Fctr.[b]	Design Tensile Strength[c] ≤ 3.5 ksi	Design Tensile Strength[c] > 3.5 ksi
≤ 1.5	Y		Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.35	$0.060\ f'_c$	$3.0\ (f'_c)^{1/2}$
			Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.35	$0.030\ f'_c$	$1.9\ (f'_c)^{1/2}$
> 1.5	Y		Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.32	$0.055\ f'_c$	$2.7\ (f'_c)^{1/2}$
			Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.32	$0.025\ f'_c$	$1.7\ (f'_c)^{1/2}$
≤ 1.5		N	Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.18	$0.030\ f'_c$	$1.5\ (f'_c)^{1/2}$
			Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.18	$0.015\ f'_c$	$1.0\ (f'_c)^{1/2}$
> 1.5		N	Max	$0.17\ f'_c$	$8.5\ (f'_c)^{1/2}$	0.16	$0.025\ f'_c$	$1.4\ (f'_c)^{1/2}$
			Min	$0.08\ f'_c$	$5.5\ (f'_c)^{1/2}$	0.16	$0.015\ f'_c$	$0.9\ (f'_c)^{1/2}$

[a] Splitting tensile strength of parent material.
[b] Includes conversion for direct tensile, joint strength, and probable percent of bonded joint.

c Direct tensile strength of construction joints.

d. While tracked dozers impart some vibration and a significant amount of compaction to the concrete, the additional vibration needed for joint bond can be attained, without over-rolling the top lift, by the placement on one plastic lift upon another, and rolling between lifts. The thickness of rolled lifts should not be more than approximately one foot.

e. Lower heat generation and water requirements can be obtained with improved workability by utilizing fly ash in place of aggregate fines in the RCC mixes. This can be accomplished at cost savings despite higher cost of fly ash due to the water-reducing qualities and additional pozzolanic strength contribution of the ash.

f. For workable mixtures and construction procedures equivalent to Elk Creek, the above design recommendations assume direct tensile strength of RCC to range from 0.73 to 1.06 times that of conventional concrete.

E-5. References

{1} Raphael, Jerome M. Tensile Strength of Concrete, ACI Journal / March-April 1984, pp 158-165.

{2} Thomas, T. C. Hsu, and Slate, Floyd O. Tensile Bond Strength Between Aggregate and Cement, Paste or Mortar, ACE Journal Proceedings V. 60 No. 4, April 1963, pp 465-485.

{3} Walker, Stanton, and Bloem, Delmar L. Effects of Aggregate Size on Properties of Concrete, ACI Proceedings V. 57, April 1960, pp 283-298.

{4} Tynes, W. O. Correlation Between Tensile and Compressive Strengths for Lean Mass Concrete, Technical Report C-74-2, U.S. Army Engineer Waterways Experiment Station Concrete Laboratory, Vicksburg, MS.

{5} Gonnerman, H. F., and Shuman, E. C. Compression, Flexural, and Tension Tests of Plain Concrete, Proceedings, ASTM, F 28, Part II, 1928, pp 527-564.

{6} Kupfer, Helmut, Hilsdorf, Hubert K., and Rüsch, Hubert. Behavior of Concrete Under Biaxial Stresses, ACI Journal, Proceedings V. 66 No. 8, Aug 1969, pp 656-666.

{7} U.S. Army Corps of Engineers, Jacksonville District. Portugues and Bucana Rivers, Puerto Rico, Design Memorandum No. 21 (Structural Properties and Special Studies) Appendix B, Portugues Dam.

{8} Cady, P. D., Clear, K. C., and Marshall, L. G. Tensile Strength Reduction of Mortar and Concrete Due to Moisture Gradients, ACI Journal Proceedings V. 69 No. 11, November 1972, pp 700-705.

{9} Dunston, Malcom R. H. Rolled concrete for dams - construction trials using high fly ash content concrete, Technical Note No. 106, Construction Industry Research and Information Association, London, May 1981, pp 96.

{10} L'Hermite, R. Recent Research on Concrete, Cement and Concrete Association Library Translation Cj 24, London, 1951.

{11} Bloem, D. L. Study of horizontal mould for concrete cylinders, Series D-58, National Ready-mix Concrete Association, Washington, DC, Sept. 1958.

{12} Johnston, C. D. Anisotrophy of concrete and its practical application, Highway Research Board 423, 1973, pp 11-16.

{13} Petersons, N. Strength of concrete in finished structures, Transactions of the Royal Institute of Technology 232, Stockholm, Sweden, 1964.

{14} Hughes, B. P., and Ash, J. E. Water gain and its effect on concrete. Concrete, December 1969 3, pp 494-496.

{15} Gilkey, H. J. Water gain and allied phenomena in concrete work, Engineering News Record, February 1927, 98(no. 6), pp 242-247.

{16} Cannon, Robert W. Compaction of Mass Concrete with Vibratory Roller, ACI Journal / October 1974, pp 506-513.

{17} ACI 207.5R-89. Roller Compacted Mass Concrete, Manual of Concrete Practice, American Concrete Institute, 1989.

{18} Elk Creek Lake Rogue River Basin, Oregon, Concrete Investigations and RCC Construction Techniques for a Concrete Gravity Dam, U.S. Army Corps of Engineers, Portland District, Design Memorandum 8, Final, 1987.

{19} General Construction Specification No. G-48 for Roller Compacted Concrete, Tennessee Valley Authority, Knoxville, Aug. 1975, Appendix A & B.

{20} Dunstan, Malcom R. H. Roller concrete for dams - a laboratory study of the properties of high fly ash content concrete, Construction Industry Research and Information Association Technical Note 105, May 1981, pp 96.

{21} Mix Design Investigation - Roller Compacted Concrete Construction, Upper Stillwater Dam, Utah, U.S. Department of Interior, Bureau of Reclamation, REC-ERC-84-15, June 1984, pp 72.

{22} Cannon, Robert W. Design Considerations for Roller Compacted Concrete and Rollcrete in Dams, ACI Concrete International / Dec. 1985, pp 50-58.

{23} Communication from Ralph Strom to Bob Cannon dated 14 March 1990. Data on tensile strengths from Willow Creek and Elk Creek.

{24} Investigation of Methods of Preparing Horizontal Construction Joints in Concrete, Technical Report No. 6-518, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, July 1959; Testing of Joints in Large Blocks, Report No. 2, July 1963; Evaluation of High-Pressure Water Jet and Joint Preparation Procedures, Report No. 4, Aug. 1973.

{25} Saucier, K. L. Investigation of No-Slump Roller Compacted Concrete (RCC) for Use in Mass Concrete Construction, Roller Concrete for Dams, International CIRIA Conference Held on 9/10 June 1981, pp 9-1 through 9-14.

{26} Cannon, Robert W. Boney Falls Dam RCC Spillway, Evaluation of Cores. Communication transmitted to Barr Engineering of Minneapolis, 9 January 1990.

{27} Dolen, T. P., and Drahushak-Crow, R. Evaluation of Cores from Two RCC Gravity Dams, ASCE Special Publication, Roller Compacted Concrete II, pp 203-213.

{28} Dolen, T. P., and Tayabji, S. D. Bond Strength of Roller Compacted Concrete, ASCE Special Publication, Roller Compacted Concrete II, pp 170-186.

{29} McLean, F. G., and Pierce, J. S. Comparison of Joint Strengths for Conventional and Roller Compacted Concrete, ASCE Special Publication, Roller Compacted Concrete II, pp 151-169.

{30} Wong, Biscoff, and Johnson. Dam Strengthening and Raising, ASCE Special Publication, Roller Compacted Concrete II, pp ?